Socket Friction Capacity of Large Diameter Drilled Shafts in Highly Weathered Rock

Capacite Frictionnelle Des Puits Grand Diameter Fores En Pıerre Fortemente Altere

Orhan Erol
Civil Engineering Dept., Middle East Technical University, Ankara, Turkey

Atilla Horoz
Yuksel Proje Co.Inc.,Ankara, Turkey

Ahmet Sağlamer
Civil Engineering Dept., Istanbul Technical University, Istanbul, Turkey

ABSTRACT
Field load test of three drilled shafts socketed in completely weathered and exteremely fractured metadiorite / amphibolite were conducted using the Osterberg load cell. The objective of the testing was to determine the uplift capacity of short sockets bored in the base rock for foundation design verification of two major viaducts being constructed in the vicinity of North Anatolian Fault. Short piles with 2, 3, and 4m socket lengths were loaded to failure to obtain the ultimate frictional capacity in tension. Observed frictional capacity of the extremely weathered rock (soil like material) was found on the order of 200 kPa. The tests allowed to observe the mobilization of end bearing stresses at the lower part of the pile without reaching to failure.

RÉSUMÉ
Site chargement tests pour trois puits pénétré en pierre complètement altérée et extrêmement fracturée metadiorite / amphibolite ont conduites utilisant l'Osterberg chargement cellule. L'objective de test était a déterminer la capacité d'élévation des pieux courts forés en base de pierre pour la vérification du design de fondation de deux viaducs majors en construction près de Nord Anatolien Faut. Les pieux courts avec la longueur de 2, 3 et 4 m ont chargées jusqu'à l'échec pour obtenir la capacité frictionnelle ultime en tension. La capacité frictionnelle observée de pierre extrêmement altérée (comme sol) était trouvé environs 200 kPa. Les tests ont permis d'observer la mobilisation de contraintes au bout sur la partie inférieure de pile sans avoir l'échec.

1 INTRODUCTION

The Anatolian Motorway Gümüşova - Gerede Section, a portion of the Trans-Europian Motorway (TEM) network that links Ankara to Istanbul, and Turkey to Europe is under construction. There are several viaducts in the Bolu Mountain Crossing being 25 km long, and follows narrow an "V" shaped Asarsuyu Valley with a longitudinal slope of about 6 percent. This part of the Motorway is within a high seismic zone, as being in the vicinity of the North Anatolian Fault . One of the constructed viaducts situated in this region, had some serious damage during 1999 Duzce Earthquake with a moment magnitude of M = 7.2. The recent re-evaluation of seismicity of the region revealed that in design of the viaducts horizontal and vertical ground accelerations of a_h = 0.56g and a_v = 0.28g must be considered (Erdik and Yilmaz 2000). Therefore relatively high magnitudes of uplift loads act on the piles of the piers which must be resisted by the rock sockets.

The base rock quality in the region is very poor to poor due to extensive seismic activities occurred over the years . Therefore design specifications required determination of uplift capacity of rock socket by pile load tests. Three pile load tests were conducted with variable socket lengths in almost identical rock conditions, and the sockets were loaded to failure to obtain ultimate frictinal capacity of the base rock. This paper discusses the testing procedures, test results and the magnitude of the frictional capacity of the particular weak rock encountered in the region.

2 GEOLOGY AND SUBSURFACE CONDITIONS

Asarsuyu Vallye stretches in the East-West direction, parallel to the North Anatolian Fault. Base rock of the region are Paleozoic aged amphibolites, amphibolic gneiss, metadiorite and metagranite. All these rock units have been subject to extensive metamorphism due to Paleotectonic and Neotectonic movements. Quaternary aged slope debris and alluviums, being 10 – 15 m thick , overlie the base rock. Faults are common at the site and heavily fractured and smashed rock zones are extensive in the region (Saglamer et al 2003).

At the location of the test piles, there is a shallow layer of river alluvium, underlain by amphibolite rock unit. The geologic description of the base rock is as follows: brwnish green, dark gray, completely weathered, exteremely weak and completely fractured metadiorite and amphibolite, fractures are filled with alcali feldspar. The RQD values of the rock is practically zero, therefore no uniaxial compressive stregth data was available. The results of point load index tests ,shown in Fig. 1, indicate that the base rock has typically point load index in the range 0.1 MPa < I_s < 0.3 MPa.

3 LOAD TESTS

The load tests were performed using Osterberg Load Cells (O-cell). The Osterberg Cell method has become useful in researching the processes involved in mobilizing the side shear and end bearing capacity of bored and driven piles. (Osterberg J.O., J.A. Hayes 2001)

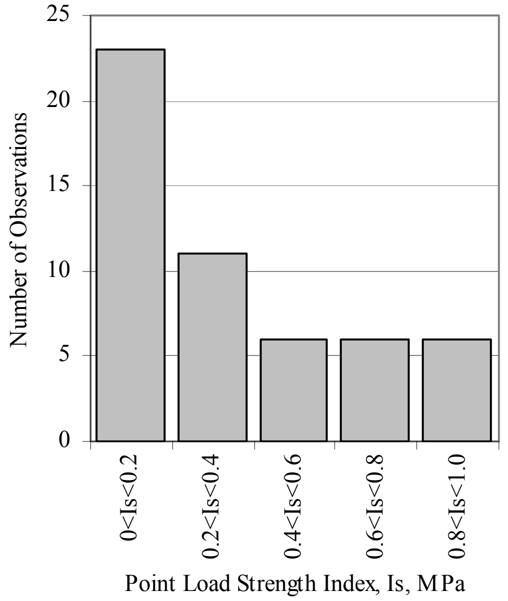


Figure 1. Point load strength Index of the rock

The Osterberg Load Cell (O-cell) is a jack like hydraulic device placed at the bottom or some distance up from the bottom. When pressure is applied to the device, an equal upward and downward force is applied to the pile. The load is determined from a load pressure calibration curve. The downward force is resisted by the side shear and therefore no overhead load frame or dead weight reaction is required. The test is continued until the ultimate in side shear ,the ultimate in end bearing or the capacity of the O-cell is reached, whichever occurs first. Since the upward and downward loads are always equal, the tested capacity is equal to twice or more the load indicated by the O-cell.

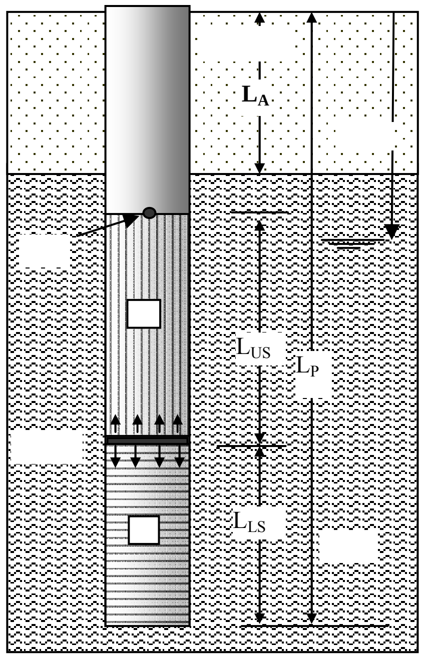
In the testing program 1200mm test piles were constructed dry to tip elevation, although ground water was noted entering the excavation. The pile was started with a 1320mm outer diameter casing. An auger was used for drilling, and a bucket was used for cleaning the pile tip.Concrete was placed by tremie through a 250mm diameter pipe into the base of the pile until the top of concrete reached the target base of O-cell elevation. After the reinforcing cage with attached O-cell and instrumentation was inserted into the pile, the pile was then tremied until the concrete reached ground elevation. The casing was removed immediately after concrete placement.The subsurface profile, the location of the O-cells, dimensions of the upper (under tension) and the lower (under compression) sockets are shown in Fig. 2.

Expansion of the O-cell was measured using two bottom plate telltales extending above the top of pile.The compression of the pile above the top of O-cell assembly was measured by means of two traditional telltales extending above the top of the pile. Bottom plate telltales were monitored with attached LVDTs attached to the top of the pile .

One level of two sister bar vibrating wire strain gages was installed in the pile above the O-cell . The strain gages were used to assess the side shear load transfer of the rock socket above the O-cell. Both Bourden pressure gages and a vibrating wire pressure transducer were used to mesure the pressure applied to the O-cell at each load increment.The load increments were applied in accordance with the procedures described in Quick Load Test Method for Individual Piles (ASTM D1143), holding each successive load increment constant for eight minutes by manually adjusting the O-cell pressure. Assembly and installation of the O-cell and the instrumentation was carried out by LOADTEST Int. Inc. (LOADTEST Int. Inc. 2001).

4 RESULTS AND DISCUSSION

The load - displacement behavior of the three upper and the lower sockets are shown in Fig.3. In Fig.3 the upper movements are shown with positive values indicating the behavior of the socket in tension, and the downward movements with negative values showing combined end bearing and side shear in compression, in the upper and the lower sockets, respectively.



LMP : LOAD MEASURING POINT

US : UPPER SOCKET

LS : LOWER SOCKET

	TEST 1	TEST 2	TEST3
L_A (m)	2.50	3.50	4.50
L_{US} (m)	4.00	3.00	2.00
L_{LS} (m)	2.84	2.35	1.19
L_P (m)	10.00	9.10	8.00
GWT (m)	3.50	3.00	5.80

Figure 2. Testing program

The three of the upper sockets were failed in tension allowing evaluation of the ultimate value of the side friction in tension, f_{st} . The failure is clearly indicated by the 35mm to 55mm residual displacements recorded upon unloading.The mobilization of f_{st} with upward movements in three tests are shown in Fig. 4. The three upper socket exhibited identical side shear vs. pile movement behavior despite the differences in the socket lengths. It is found that the ultimate side shear resistance is 210 kPa when an average value of the three tests is considered. The major portion of the side shear (e.i. 90 percent of ultimate value) is mobilized at a displacement of 10mm which corresponds to a value slightly less than one percent of the pile diameter.

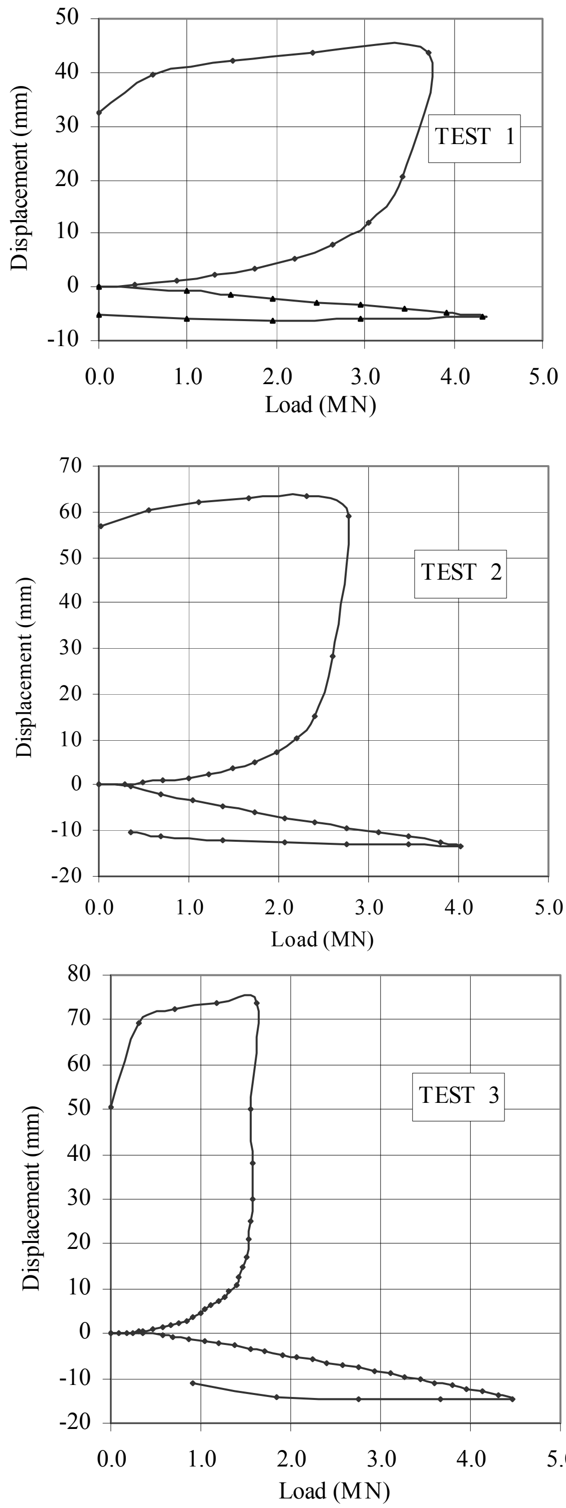


Figure 3. Load Settlement behavior of the sockets

It is noted that mobilization pattern of the side shear in tension for the decomposed rock considered, shown in Fig. 4 , does not matches with the behavior of rock sockets in sound rock where the peak shear resistance is mobilized at smaller relative displacements (e.i. on the order of a mm) followed by a immediate reduction in shear resistance reaching to residual strengths (Wyllie et al 1992). Thus the particular decomposed rock shows a soil-like behavior rather than a sound rock.

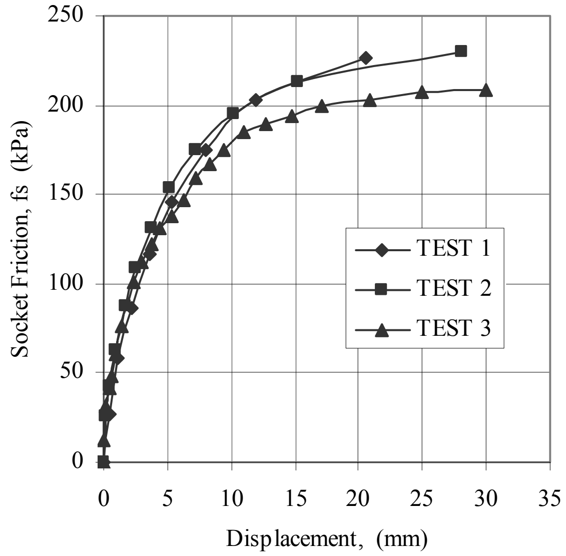


Figure 4. Mobilization of side shear in tension

The average point load index value of the particular rock is on the order of I_s = 0.35 MPa, suggesting an uniaxial compressive strength of q_u = 8.4 MPa for the base rock. Then the ultimate side shear resistance of the base rock measured in the tests is only about 2.5 percent of the anticipated compressive strength, which is a range much smaller than the suggested values by the proposed empirical correlations between f_{st} and q_u (Williams et al 1980).

There is general expectation that side shear in tension will be les than side shear in compression for two reasons: i. the Poisson's ratio effect in a favorible direction (increases horizontal stresses against pile shaft) when loaded in compression vs.unfavorible (decrease lateral stresses) when loaded in tension; and ii. accumulated side shear in tension tests reduces the underlying effective stresses while the side shear in a compression test , acting in the opposite direction increases underlying effective stresses. These decreases and increases may then translate into a similar change in horizontal stresses and therefore side shear stresses. However the ratio of side shear in tension to compression is close to one in cohesive materials and slightly less than one in granular material.

In the interpretation of the test results to evaluate the base resistance of the piles, it is assumed that the magnitude of side shear in tension and compression are the same for practical purposes. Thus the O-cell data allowed to evaluate mobilization of end bearing compressive stresses by subtracting the lower socket side shear load from the total downward reaction measured during the test.

The typical test results are shown in Fig. 5 for two different socket lengths. The data trends indicate that load transfer to the pile base is an immediate process and end bearing is mobilized as soon as the load is applied to the lower socket, in the test with relatively short sockets (e.i. 1.2m) . However in the longer socket (e.i. 2.4m) the load is not transferred to the pile base untill a relative downward displacement of about 5mm is reached, and then the the base resistance follows a similar a similar mobilization pattern with the shorter socket (e.i. paralel lines in Fig. 5). The ultimate end bearing resistance of the piles were not reached at downward movements of about 15mm, where the load-displacement behavior is still in the elastic range (linear).

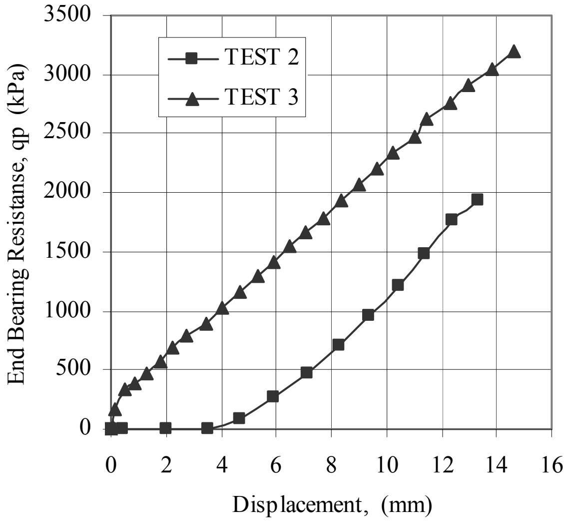


Figure 5. Mobilization of end bearing

5 CONCLUSIONS

Field load test of three drilled shafts having diameter of 1.2m, socketed in completely weathered and exteremely fractured metadiorite / amphibolite were conducted using the Osterberg load cell. Socket lengths were 2,3 and 4m. The following conclusions are derived from the test results:

The particular completely decomposed rock exhibits a soil like behavior, and the side shear resistance of the sockets in tension is much less than the suggested values by the proposed empirical correlations between socket resistance and uniaxial compressive strength.

The ultimate side shear resistance of the particular rock material is 210 kPa in tension. The major portion of the side shear (e.i. 90 percent of ultimate value) is mobilized at a displacement of 10mm which corresponds to a value slightly less than one percent of the pile diameter.

The ultimate end bearing resistance of the piles were not reached at downward movements of about 15mm, where the load-displacement behavior is still in the elastic range (linear).

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REFERENCES

ASTM D1143 Standard Test Method for Piles Under Static Axial Load.

Erdik M., Yilmaz C., 2000 "Reassessment of site specific probabilistic seismic hazard and fault rupture hazard for Viaduct 1 of Gumusova-Gerede Motorway, Bosphorus University, Istanbul, Turkey.

LOADTEST Int.Inc. 2001 " Report on drilled pile load testing (Osterberg Method) " Unpublished report No: 45.110/2D/VI4/GT 002

Osterberg J.O., J:A:Hayes 2001, "The Osterberg Load Cell as a Research Tool" Proc. Of the XVth ICSMGE, Vol.2, pp:977-979, İstanbul, Turkey).

Saglamer A., Yilmaz E., Erol O., 2003 " Compression and tension capacities of rock socketed drilled piers" Proc. XIII ECSMGE, Vol.2, pp:359-364, Prague.

Williams, A.F., Johnson E.W., Donald I.B. 1980 " Design of socketed piles in weak rock" Proc.of Int. Conf. On Structural Foundations on Rock, Sydney, Australia.

Wyllie D.C. , 1992 Foundations on Rock , Chapman and Hall.